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PILE LOADING TESTS, MORGANZA FLOODWAY CONTROL STRUCTURE

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PILE LOADING TESTS

MORGANZA FLOODWAY CONTROL STRUCTURE

Charles I. Mansur* and John A. Focht, Jr.**

SYNOPSIS

A comprehensive pile testing program was undertaken at the site of the Morganza Floodway control structure to determine the size of pile required to carry the design compression and tension loadings without any significant movement. As 80 ft of very compressible clay overlie sand at the site of the structure, an attempt was made to obtain the bearing capacity of only the portion of the piles penetrating into sand. This was accomplished by testing different size compression piles in pairs. One pile of each pair which was stopped above the top of the sand was loaded to obtain the resistance developed by skin friction in the clay. A test on a second pile driven into the sand indicated the total supporting capacity of both the clay and sand. The difference in the capacities of these two piles was then taken as the tip capacity of the pile driven into sand. Tension tests were made on a third pile of the same size which was driven into the sand. The sizes of the piles tested ranged from a tapered pile with an 8-in. tip to a 30-in. pipe pile. Details of the loading arrangement and test procedures are described.

From the results of the tests a 20-in.-diameter pile was selected to carry the structure loads of 100 tons in compression and 25 tons in tension, with an ample factor of safety against both detrimental settlement and sudden plunging. It was found that the skin friction developed by both compression and tension piles in clay was very nearly the shear strength of the clay as indicated by unconfined compression tests. The bearing capacities of the portions of the piles in sand, as computed by several different methods suggested for piles, showed reasonable agreement with the pile tests results. The pile capacities, computed by dynamic pile-driving formulas did not, in general, check the pile test results.

Introduction

The soil conditions at the site for the Morganza Floodway control structure necessitated the use of long piles driven through soft clay into sand at a depth of approximately 80 ft. As the pile foundation for the structure would be required to carry rather large loads in both compression and in tension with very little movement, it was decided to perform a series of pile loading tests to determine the size of pile required to carry a design load of 100 tons without any significant settlement. The purpose of this paper is to summarize the procedures used in making the pile tests, give the types of piles tested, and present

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the results and conclusions obtained. Comparisons are also made of test results with values computed from various pile capacity formulas and from pile driving formulas.

Description of structure

The Morganza Floodway control structure is located about three miles northwest of Morganza, Louisiana. The structure is to consist of a gated weir for control of flood waters combined with a railroad and highway crossing of the floodway. The structure, approximately 4000 ft in length, is connected to the guide levees of the floodway by combined embankments which carry the highway and railroad and serve as a portion of the main-line levee system of the Mississippi River. The pile foundation beneath the gated weir consists of precast concrete piles driven to sand on a 2 on 1 batter. There are 27 piles beneath each pier; 11 battered upstream and 16 downstream.

Foundation conditions

The soil conditions at the structure site consist of strata of predominantly highly plastic clays about 80 ft thick underlain by sand. The clay strata have average water contents ranging from about 42 to 60 per cent and liquid limits ranging from 60 to 100.

Consolidation tests on samples of clay from beneath the structure showed that the foundation is very compressible and that piles driven to sand would be necessary to support the structure. Existing embankments, 30 ft high, near the structure have settled 2-1/2 to 5 ft.

Unconfined compression and unconsolidated undrained triaxial compression tests on undisturbed samples from the clay strata beneath the structure indicated an over-all average cohesive shear strength of 660 lb per sq ft for the foundation clays. Six unconfined compression tests on remolded samples taken at varying depths beneath the structure showed an average remolded strength of 620 lb per sq ft. Thus, pile-driving operations were not expected to influence the shear strength of the foundation clays appreciably.

The sand underlying the clay strata is a uniformly graded fine sand composed of subrounded to subangular grains. Driving of a standard 1-3/8-in. I.D. split-spoon sampler with a 30 in. drop of a 140-lb hammer showed the sand to have an average driving resistance of about 55 blows per ft. The relative density of the sand was about 80 per cent.

Purpose of pile tests

The purpose of the pile loading tests was to determine what size pile would be required to carry a 100-ton compression load and what tension load that size pile would carry. All test piles were driven vertically for ease of testing. Although the tip bearing capacity of vertical piles is probably somewhat greater than for the batter piles, the difference probably does not amount to more than 5 to 15 per cent. Detailed reports on these loading tests were prepared by the New Orleans District, CE[1], and the Waterways Experiment Station[2] in December 1949 and January 1950.

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1. Morganza Control Structure Pile Tests, New Orleans District, CE, New Orleans, La.
2. Pile Loading Tests - Combined Morganza Floodway Control Structure, T.M. 3-308, Waterways Experiment Station, CE, Vicksburg, Miss.

General Test Procedures

As the clay overlying the sand is very compressible, the compression tests were made in such a manner that the bearing capacity of the portion of the piles driven into the sand could be evaluated. For the purpose of simplicity, that portion of the pile driven into sand is hereafter called the "tip"; the point and the sides of the pile in sand are both included in this definition and both carry part of the load. To determine the bearing capacity of the tip it was necessary to eliminate completely or to measure the frictional resistance developed along that portion of the pile in the clay stratum. In these tests the resistance developed in the clay was determined by testing the piles in pairs. The frictional resistance in the clay stratum was determined from a pile stopped about 5 ft above the top of the sand; the combined frictional resistance and tip capacity were determined from a pile driven into the sand. Piles stopped in clay are called "a" piles in this paper, whereas piles driven into sand have been termed "b" piles. The tension piles also were driven into sand. As all piles beneath the structure were to be loaded in compression prior to any tension loading, all tension test piles were loaded with a normal "working" compression load prior to testing in tension. Holes were excavated at each test pile location to eliminate any skin friction in material above the proposed base of the structure.

Types of Piles and Driving Equipment

The types and sizes of piles tested in compression at locations along the length of the structure are shown in fig. 1 and in the following tabulation.

Type of Pile	Size and Description
Cylindrical pipe	3/8-in. wall thickness 18-in., 24-in., 30-in. diameters 1.5-ft, 1.9-ft, 2.4-ft long conical points
Monotube, uniform tapered	No. 7 gage metal 18-in. butt diameter, tapered 1 in. in 7 ft to 8-in. diameter hemispherical tip
Monotube, constant section	No. 3 and No. 7 gage metal 12-in. diameter, 9-in. long conical point
Precast concrete	22 in. square, point tapered to 8 in. in 2 ft

A tension test was conducted on each pile with the exception of the 30-in. diameter pipe pile and the precast concrete pile.

The piles were driven by a skid-mounted, whirly-type driver with 82-ft leads. A Vulcan No. OR hammer was used to drive the 24- and 30-in. pipe piles and the 22-in.-square concrete piles. A Vulcan No. 1 hammer was used for driving all anchor piles, the Monotube piles, and the 18-in. pipe piles. All piles were filled with concrete a few days after driving. A period of 4 to 6 weeks elapsed after driving before any loads were applied to the piles.

Compression Test Piles

Load Arrangement

The test piles were loaded by jacking against concrete weights stacked on platforms over the test piles as shown on fig. 2. The platforms were supported

on six timber piles in a row on each side of the test pile. The weights were precast concrete slabs and rectangular blocks weighing approximately 3.5 and 5 tons, respectively. Hydraulic jacks seated on fabricated steel pedestals placed on top of the test piles were used to apply the loads to the piles. The jacks used were two 200-ton, six 100-ton, and four 60-ton capacities.

Loading procedure

"a" piles. The "a" piles, which were stopped in clay, were loaded and tested to failure prior to loading the "b" piles which were driven into sand. In general, loads were applied to the piles in 20-ton increments at the rate of 1-1/2 tons per minute. Each increment was held until 80 per cent of the estimated ultimate movement under that increment of load had taken place, but not for longer than 24 hours. Time-settlement curves were plotted during the tests to estimate the ultimate settlement under each increment of load. The 80- and 160-ton loads were maintained for 24 hours or until movement practically stopped, whichever period was longer. After completion of the 40-, 80-, and 160-ton loads and at the end of the test, the piles were unloaded and allowed to rebound until 80 per cent of the estimated movement occurred, but not for periods longer than 12 hours. Unloading and reloading were done at the rate of 1-1/2 tons per minute. All "a" piles were loaded to failure.

"b" piles. The "b" piles, which were driven into sand, were loaded by the same loading schedule used for the corresponding "a" piles. When the load was 60 tons more than the maximum load of the corresponding "a" pile, the load was removed and the pile allowed to rebound. The pile then was loaded again and when the load became 100 tons greater than the maximum load of the "a" pile it was held constant until practically all movement stopped. The load was then removed and the pile allowed to rebound. Loading was then resumed; when the load reached 160 tons greater than the maximum load of the "a" pile the load was removed and rebound of the pile permitted. Loading then was resumed up to a load of 200 tons more than the maximum load of the "a" pile. This load was maintained until movement had practically stopped at which time it was removed and the pile allowed to rebound. All "b" piles except the 30-in. pipe pile were loaded to failure. This pile successfully carried 400 tons without failure. Rebound was allowed on all piles at the end of the test.

Measurement of movement

The movement of the pile under load was measured by means of an engineers level and dial deflection gages. The gages were mounted on opposite sides of the pile and measured the movement with reference to concrete slabs 12 to 14 ft away from the test pile.

Test data

Typical test data for compression piles C-4-a and C-4-b are shown graphically on fig. 3 and 4.

Load settlement curves. A curve of gross movement of the pile head versus pile load was developed from the test data on each pile. The points representing loads which were held for 24 hours were given primary consideration in drawing the curves shown on fig. 5 and 6. This procedure results in a curve which should represent (up to the maximum load) the movement of the pile head under sustained load, neglecting consolidation of the clay foundation.

Net and elastic settlement curves. As each test pile was allowed to rebound under no load a number of times during the test, it was possible to determine the net and elastic settlement curves for each pile. The points representing the net settlement remaining after allowing rebound and the amount of elastic rebound are plotted for each rebound for each pile on fig. 5 and 6.

Tip-load curves. A curve approximating the load carried by the tip of the "b" pile (the portion of the pile in the sand) was computed for each "b" pile by subtracting at any gross settlement the "a" pile load from the "b" pile load. This method is not exactly correct, as the load carried by a friction pile is not distributed to the foundation in the same way as the load on a point bearing pile because, at similar movements of the top of the pile, the distribution of strain is different along the two piles. However, no other method of developing a tip-load deflection curve from this type of test was considered to be any more accurate.

Pile failure loads

The determination of the "failure load" of a test pile is usually arbitrary. Therefore, various procedures were utilized to obtain a value for each pile. The methods used for selecting a failure load for each compression pile were as follow:

a. Determine the load which produced a net settlement of 0.25 in.

b. Determine the load indicated by the intersection of tangent lines to the flatter portion of the gross settlement curve and to the steeper portion of the same curve.

c. By inspection determine the load beyond which there was an increase in gross settlement disproportionate to the increase in load.

d. Determine the load at which the slope of the net settlement curve is four times the slope of the elastic deformation curve.

e. By inspection determine the load beyond which there was an increase in net settlement disproportionate to the increase in load.

The results of the analysis of each test are given in table 1 with the average of the five methods of determinations. The average of the five methods is used in all further analyses. The 30-in. pipe pile driven into sand, C-6-b, did not fail under a maximum load of 400 tons. However, a failure load was estimated for this pipe by extrapolating the settlement-load curve.

Other procedures for determination of the failure load for a test pile which have been suggested in the past or are given in building codes were discarded after preliminary trials. The methods not utilized are primarily for use with tests on smaller piles under much lighter loads.

There are no significant differences in the values indicated for any pile by the five methods. If a pile test is not being performed under a specific building code with a definite method established for determination of the failure load, it appears desirable, and at least helpful, to average the results indicated by several methods of analysis.

Average bearing capacity of pile tip

As the piling was to be designed so that the portion of the pile in the sand would carry the design load, the average relation of the bearing capacity of the pile tip to the diameter of the pile tip was determined. Two methods were

utilized to compute the relation. In the first the difference between the average failure loads of the "a" and "b" compression piles, as listed in table 1, was taken as the bearing capacity of the tip of the "b" piles. The second method utilized the tip failure load as determined by inspection of the tip load curve. The capacity of the tip of each "b" pile as determined by each of the two methods is tabulated in table 1 and is plotted versus the tip diameter on fig. 7. Each of these methods indicates an approximate straight-line relation between the tip load and tip diameter. Since the penetration into sand of all piles was not the same, a direct comparison was not possible with the approximately parabolic relationship between load and tip diameter indicated by most bearing-capacity formulas for piles driven into sand foundations.

Average skin friction in clay

The skin friction developed by the compression piles stopped in clay above the sand at average failure loads was computed using the nominal diameter of the piles and the embedded length of the pile. The resulting values were as follow:

Pile	Type of Pile	Diameter of Pile Inches	Skin Friction Lb/Sq Ft
C-1-a	Pipe	24	730
C-2-a	Monotube, tapered	8 (tip)	694
C-3-a	Monotube, constant section	12	543
C-4-a	Pipe	18	524
C-5-a	Pipe	24	660
C-6-a	Pipe	30	650
C-7-a	Precast concrete	22 (square)	589
	Average	--	630

The average value of 630 lb per sq ft checks closely the average shear strength of 660 lb per sq ft as determined by shear strength tests on undisturbed samples from the foundation borings.

Required size of pile

The total capacity of a pile at this site as regards plunging into the ground is the sum of the capacities developed in the clay by skin friction and in sand by skin friction and point resistance. This capacity was measured by the piles driven into sand, type "b". A pile of sufficient circumference could be driven which would not penetrate into the sand and yet would have an adequate total capacity. However, under the sustained load, such a pile would undergo considerable settlement due to consolidation within the clay. The load on a pile driven into sand initially will be carried in a large part by skin friction in the clay. However, as the clay at the site is quite compressible, the clay will tend to be relieved in time of almost all of its supporting stress by consolidation within the clay strata. If the total load applied to the pile is greater than the capacity of the portion of the pile in sand, the pile will not fail by plunging but will settle appreciably due to consolidation. Thus, to guard against detrimental settlements, the long-time capacity of the piles was taken as only the capacity of the tip of the piles in sand. On the basis of the data shown on fig. 7, a pile diameter of 20 in. was selected as being adequate to carry the design compression load of 100 tons with a factor of safety of approximately 1.5 against detrimental settlement, and a factor of safety of approximately 3.0 against failure by plunging. Octagonal piles with a minimum dimension of 20 in. were

used for lengths up to 100 ft; 20-in. square piles were used where lengths greater than 100 ft were required.

Bearing capacity formulas

The results of the pile tests were utilized to compute the indicated angle of internal friction of the sand, using methods that have been suggested for computing the bearing capacity of piles or deep piers. The basic methods used were: (a) the bearing capacity equation for cylindrical piers and piles, by Terzaghi and Peck[3]; (b) an approach suggested by Terzaghi for deep piers[4]; and (c) an equation by Jaky for piles[5] (see fig. 8). Other methods were used in preliminary analyses but were discarded as they indicated absurd values. The angle of internal friction of the sand was computed by each basic method and modifications thereof for each compression pile driven into sand using the indicated capacity of the pile tip, less the skin friction in the clay below the tip of the "a" piles and the top of sand, and the actual penetration of the pile in sand. To determine the length of penetration of the pile into sand it was assumed, from the nature of the pile driving records, that when the driving resistance reached 30 blows per ft the top of the tapered point was at the upper surface of the sand.

The angle of internal friction of the sand necessary to give the corrected bearing capacity was obtained by a trial and error method using the formulas given in fig. 8.

Indicated friction angle of sand The angle of internal friction of the sand as computed from the corrected tip load carried by each "b" pile is listed in table 3. With the exception of Method "C" the analyses show that the friction angle of the sand is between 30° and 35° on the average. This is in the range of the friction angles usually obtained for this type of sand.

Discussion

The computed angle of internal friction of the sand is about the same, whether computed from Method "B" or Method "E", neither of which take credit for any skin friction in the sand, or from Methods "A" and "D" which assume skin friction of the sand against the pile to be acting. As it is believed that skin friction of the sand on the pile will carry some of the tip load, Method "A" and "D" appear to be the most logical for estimating the tip bearing capacity of piles driven into sand.

A usual assumption for the shear strength of sand is $\phi = 30^{\circ}$. Using this value, the bearing capacities of various size piles driven 5 ft into sand with an overburden of 85 ft have been computed using the methods given in fig. 8 and are shown as curves in fig. 9. The measured tip bearing capacities of the piles, adjusted to the same conditions assumed for computation of the curves are also

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3. Terzaghi and Peck, "Soil Mechanics in Engineering Practice," (New York: John Wiley and Sons, 1948), pp. 176-177.
4. Karl Terzaghi, "Theoretical Soil Mechanics," (New York: John Wiley and Sons, 1934), p. 134, and discussion by A. Casagrande and R. E. Fadum of paper, "Application of Soil Mechanics in Designing Building Foundations," Transactions, American Society of Civil Engineers, vol. 109 (1944), p. 430.
5. J. Jaky, "On the Bearing Capacity of Piles," Proceedings of Second International Conference on Soil Mechanics, vol. I, p. 102.

shown on this figure. From the data presented it appears that Method "A" (Terzaghi and Peck) and Method "D" (Jaky) fit the test data reasonably well.

Tension Test Piles

Load arrangement

The tension piles were loaded by jacking against six timber anchor piles in a row on each side of the pile. An 8-in. I-beam was embedded about 9 ft into the concrete filling the pile shell to make the tension connection as shown in fig. 10. Two 100-ton hydraulic jacks were used to apply load to the test piles.

Loading procedure

Compression loading. As the piles beneath the structure will be subjected to a compression load prior to any tension load, piles T-1, T-2, T-4, T-5, and T-6 were subjected to a compression load of 85 tons, and pile T-3 to a load of 50 tons prior to testing in tension. The loads were applied at the rate of 1-1/2 tons per minute and were maintained for 12 hours, after which the loads were removed at the rate of 1.0 ton per minute.

Tension loading. The pull was applied to the test piles in 8-ton increments at the rate of 1.0 ton per minute. Each increment was maintained until 80 per cent of the estimated movement of the top of the pile had taken place, but for not more than 12 hours. Loads of 24, 72, and 96 tons were maintained for 24 hours or until movement practically stopped, whichever was longer. The 48-ton load was held for 48 hours on all piles except T-5; the 48-ton load on pile T-5 was held for 7 days (168 hours). After completion of the 24, 48, 72, and 96 ton loads on pile T-5 and at the end of the test, rebound was allowed until 80 per cent of the estimated movement had occurred, or for no longer than 12 hours. Unloading and reloading were done at the rate of 1.0 ton per minute. All piles except T-5 and T-6 were loaded to failure; the limit of the loading equipment was reached before these two piles could be failed.

Test data

Typical test data for tension pile T-4 are shown graphically on fig. 11.

Load-rise curves. A curve of gross movement of the pile head was developed from the test data on each pile. In drawing the curves shown on fig. 12, the points representing loads held for 24 hours were given primary consideration. Thus the resulting curve should represent (up to the maximum load) the movement of the pile head under sustained load (neglecting expansion of the clay foundation). The true shape of the curve after the maximum load is not definitely known and therefore the curves beyond the maximum load are dotted. No attempt was made to extrapolate the remainder of the curves for piles T-5 and T-6 which were not failed, as most of the other piles failed rather suddenly.

Pile failure loads

In studying the load-rise curves on fig. 12, two different conditions of failure were considered. The first is that the rise of the piles during loading would be such as to have a detrimental effect on the structure. A gross rise of 0.25 in. was taken to be the limiting criterion for this condition which was used to determine the allowable tension load. The second condition is the complete failure of the pile under load, causing a reduction in the ability of the pile to carry the load. The failure load for the second condition was determined by

an inspection of the gross and net rise curves for the load beyond which there was an increase in deflection disproportionate to the increase in load. The second condition was used to determine the average tension skin friction in the clay. The results of the analysis of each test are presented in table 4.

Average load capacity

As the embedded lengths varied somewhat, the loads which produced a gross rise of 0.25 in. were corrected to an embedded length of 75 ft by assuming that the tension capacity was directly proportional to the length. These corrected values, listed in table 4, are total loads applied to the head of the pile. These loads are also plotted against the average pile diameter on fig. 13. The tapered Monotube pile was plotted at a diameter of 13 in., the average of the tip and butt diameters, which were 8 and 18 in., respectively. The average line drawn through the points indicates that a 20-in.-diameter round pile 75 ft long can carry the design tension load of 25 tons with a factor of safety of approximately 4.5.

Average skin friction in clay

It is not possible to compute directly from the test data on the tension piles the skin friction developed in the clay. The tension piles were driven into the sand, and the sand should have a materially different skin friction on the pile than the clay. The total force carried by the clay was computed by subtracting from the pile failure load determined by inspection of the net curve the buoyant weight of the pile and the total force developed by friction in the sand, assuming a coefficient of earth pressure = 1.0 and an angle of internal friction of the sand of 30°. The unit skin friction in the clay was computed from this corrected failure load, using the embedded length in the clay and the nominal diameter of the pile. The resulting values are tabulated in table 5. For purpose of comparison the skin friction as determined from adjoining "a" compression piles is also shown in table 5. As piles T-5 and T-6 were not loaded to failure, the skin friction values listed for them were the values developed under the maximum load. The average skin friction shown in table 5 was based on the results from piles T-1 through T-4. From the data in table 5 it is concluded that the skin friction developed by either a tension or compression pile in the Morganza clay at failure is fairly close to the average strength of the foundation clays as indicated by unconfined and unconsolidated undrained triaxial compression tests.

The fact that the tapered Monotube pile T-2 indicated the highest skin friction is not believed to be highly significant, because the variations in skin friction indicated by the various types of piles could be due to variations in soil strengths and not necessarily to characteristics of the piles. From the one test made it appears that at the site tested a pile with a small taper, such as 1 in. in 7 ft for piles C-2 and T-2, will respond to loading either in tension or compression in a manner similar to a pile of constant section with the same average diameter.

Dynamic Pile Driving Formulas

Formulas investigated

Six dynamic pile driving formulas were used to compute the capacity of the compression piles driven into sand. These formulas as given below include the intended factors of safety as indicated. The symbols used are grouped after the formulas.

Engineering-News	$R = \frac{2\,W\,h}{s + 0.1}$	(F.S. = 6)
Engineering-News (modification 1)	$R = \frac{2\,W\,h}{s + 0.1\,\frac{P}{W}}$	(F.S. = 6)
Engineering-News (modification 2)	$R = \frac{2\,W\,h}{s + 0.3}$	(F.S. = 6)
Eytelwein (modified)	$R = \frac{2\,W\,h}{s + 0.3\,\frac{P}{W}}$	(F.S. = 6)
Canadian National Building Code	$R = \frac{4\,W\,h\,n\,0.9}{s + 1/2\,C}$	(F.S. = 3)

$$n = \frac{W + 0.5\,e^2\,P}{W + P}$$

$$C = \frac{3\,R}{A}\left(\frac{12\,L}{E} + 0.0001\right)$$

Pacific Coast Uniform Building Code

$$R_u = \frac{12\,Wh\,\frac{W + KP}{W + P}}{s + \frac{12\,R_u\,L}{A\,E}} \qquad \text{(F.S. = 1)}$$

K = coefficient; 0.25 for steel piles; 0.10 for all other piles

F.S. = 4 recommended by Pacific Coast Code

A = average cross-sectional area of driven parts in sq in.

C = coefficient of restitution; 0.5 for ram striking steel anvil on steel or concrete piles; 0.4 for ram striking steel anvil containing a wood cushion

E = modulus of elasticity of pile material

h = height of free fall of ram in ft

L = length of pile in ft

P = weight of driven pile in lb

R = design-carrying capacity of pile in lb

R_u = ultimate carrying capacity of pile in lb

s = set per blow in in.

w = weight of falling mass in lb

The Canadian National Building Code and the Pacific Coast Uniform Building Code are modifications of the Hiley formula. It was not possible to use the Hiley formula, as the rebound of the pile during driving was not measured. The driving data of the piles stopped above the sand ("a" piles) were not used to compute the indicated capacity by the driving formula. It is usually accepted that pile-driving formulas are meaningless for friction piles driven in clay.

Relation of formulas to test results

The driving resistance for the last full 3 in. of penetration was used in each formula to compute the design capacity of each of the compression piles driven into sand. These values are given in table 6 together with certain computed ratios of formula capacity to test capacities. It appears in general from the data presented that the pile-driving formulas did not give capacities which checked either the total or tip capacity of the test piles at the site of the Morganza floodway control structure. The only good agreement obtained was between the "ultimate capacity" as computed from the Pacific Coast formula and the "ultimate tip test capacity." In connection with the average ratios shown in table 6 it is pointed out that formulas which give ratios greater than 1 are on the radical side, whereas formulas which give ratios less than 1 are on the conservative side. The ultimate capacity indicated by Engineering-News formula and its modifications and Eytelwein formula were considerably more than either the total capacity of the pile or the capacity of the tip alone (ratios of 1.5 to 4.0). The formulas of the Canadian National and the Pacific Coast Uniform Building codes indicated conservative ultimate capacities, as the ratio of code capacity to test capacity ranged from 0.4 to 1.0.

Summary

Piles with a minimum diameter of 20 in. were recommended for the foundation of the structure to carry a load of 100 tons with little or no settlement. Octagonal piles were used for lengths up to 100 ft, and square piles were used for lengths greater than 100 ft.

The skin friction that can be developed by a friction pile in the clay at this site is nearly the same in tension as in compression, and is approximately equal to the average strength of the clay as indicated by unconfined compression and unconsolidated-undrained triaxial compression tests on undisturbed samples of the clay.

The average angle of internal friction in the foundation sand into which the piles were driven is estimated from analysis of the pile loading test data to be between 30° and 35°, which values are considered reasonable for this type of sand.

Method A, Terzaghi and Peck, and Method B, Jaky, are believed to be the most reasonable theoretical methods of those considered for use in estimating the capacity of a pile tip driven into sand, in that they allow credit for the frictional resistance of the sand along the side of the tip. All methods considered are rather empirical and, while useful for estimating purposes, they should not replace load tests. This is borne out by the scattered results and lack of complete agreement of the pile tests with the capacities indicated by theoretical methods.

In general, capacities of piles computed by the various dynamic pile-driving formulas tried did not check either the ultimate total test load or ultimate tip load, or the design total or tip loads.

Conclusions

Skin friction that can be developed by a friction pile in clay in this area may be estimated satisfactorily by using the average strengths indicated by unconfined or unconsolidated-undrained triaxial compression tests on undisturbed samples of the clay.

A reasonable estimate of the bearing capacity of a pile driven into deep, relatively dense sands may be made by using either Method A or D and an average angle of internal friction in the sand of 30^{o}. The results obtained should be used only for estimating purposes.

The use of dynamic pile-driving formulas is not warranted for long, large-diameter piles to be heavily loaded.

Acknowledgment

The pile loading tests described were carried out under the general direction of the Engineering Division of the Mississippi River Commission, CE. The layout of the testing program and interpretation of the test data were made by the Soils Division of the Waterways Experiment Station, CE. The pile tests were performed by the New Orleans District, CE.

Table 1

COMPRESSION TEST PILE

FAILURE LOADS

Pile	Size and Type	Method of Failure Load Determination: 0.25-in. Net Settlement Tons	Tangent Intersection Gross Curve Tons	Inspection of Gross Curve Tons	Slope of Net 4 times Slope of Elastic Tons	Inspection of Net Curve Tons	Average of Five Methods of Interpretation	Inspection of Tip Load Curve for Failure Load Tons
C-1-a	24-in. pipe	134	131	125	132	130	130	---
C-1-b		357	360	320	347	330	343	---
Diff.		223	229	195	215	200	213	185
C-2-a	8-in. tip Monotube tapered	80	78	78	76	73	77	---
C-2-b		169	170	145	164	155	162	---
Diff.		89	92	67	88	82	85	80
C-3-a	12-in. Monotube Cons. Sect.	60	57	56	57	55	57	---
C-3-b		149	143	145	148	140	145	---
Diff.		89	86	89	91	85	88	90
C-4-a	18-in. pipe	89	78	80	85	75	81	---
C-4-b		254	258	230	240	240	244	---
Diff.		165	180	150	155	165	163	165
C-5-a	24-in. pipe	139	135	137	185	136	136	---
C-5-b		304	307	305	302	300	304	---
Diff.		165	172	168	167	164	168	185
C-6-a	30-in. pipe	174	173	165	160	160	166	---
C-6-b*		450	462	450	455	445	452	---
Diff.*		266	289	285	295	285	286	280
C-7-a	22-in. square pre-cast conc.	139	138	135	130	135	135	---
C-7-b		293	290	285	283	285	287	---
Diff.		154	152	150	153	150	152	140

* C-6-b was not loaded to failure; values indicated are based on estimated curves.

Table 2

ADJUSTED TIP FAILURE LOADS

Pile	Size and Type	Tip Failure Load Tons (1)	Corrected Tip Failure Load Tons (2)	Adjusted Tip Failure Load Tons (3)
C-1-b	24-in. pipe	185	176	160
C-2-b	8-in. tip Monotube Tapered	80	74	62
C-3-b	12-in. Monotube constant section	90	85	101
C-4-b	18-in. pipe	165	159	173
C-5-b	24-in. pipe	185	171	163
C-6-b	30-in. pipe	280	276	275
C-7-b	22-in. square precast concrete	140	120	115

(1) Tip failure load determined by inspection of tip load curve.

(2) Correction made to loads indicated by tip load curves on basis of greater length in clay of "b" piles than "a" piles.

(3) Adjustment of corrected loads made to 5-ft penetration into sand and 85-ft depth of point using Terzaghi A equation.

Table 3

INDICATED INTERNAL FRICTION OF SAND

Pile	Size and Type	Method of Analysis				
		Terzaghi				Jaky
		Method A	Method B	Method C	Method D	Method E
		Indicated $\phi°$				
C-1-b	24-in. pipe	29	32	22	30	34
C-2-b	8-in. tip Monotube	36	39	27	37	41
C-3-b	12-in. Monotube constant section	36	36	30	37	38
C-4-b	18-in. pipe	34	35	28	36	37
C-5-b	24-in. pipe	29	31	23	30	32
C-6-b	30-in. pipe	30	32	24	31	33
C-7-b	22-in. square precast concrete	18	27	14	20	28
Avg		30	33	24	32	35

Note: Strength computed from indicated tip failure loads less the skin friction in the clay below the bottom of the "a" pile and the top of the sand

Table 4

TENSION TEST PILE FAILURE LOADS

Pile	Size and Type	Determination of Failure Load			
		0.25-in. Gross Rise Tons	Gross Rise* Tons	Inspection of Curve Gross Rise Tons	Inspection of Curve Net Rise Tons
T-1	24-in. pipe	133	139	134	134
T-2	8-in. tip Monotube tapered	80	75	125	122
T-3	12-in. Monotube constant section	53	52	88	87
T-4	16-in. pipe	117	117	125	125
T-5	24-in. pipe	150	141	200	200
T-6	24-in. pipe	167	134	160	160

*Load for 0.25-in. gross rise corrected to an embedded length of 75 ft.

Table 5

CORRECTED SKIN FRICTION IN CLAY

TENSION TEST PILES

Pile	Size and Type	Tension Skin Friction Lb/Sq Ft	Compression Skin Friction Lb/Sq Ft
T-1	24-in. pipe	608	730
T-2	8-in. tip Monotube tapered	829	694
T-3	12-in. Monotube	706	543
T-4	18-in. pipe	639	523
T-5	24-in. pipe	651	659
T-6	24-in. pipe	350	588
Average		695	623
Average of tension and compression tests		660	

Notes:

Average based on piles T-1 through T-4.

Average skin friction of piles C-1-a through C-7-a = 630 lb/sq ft.

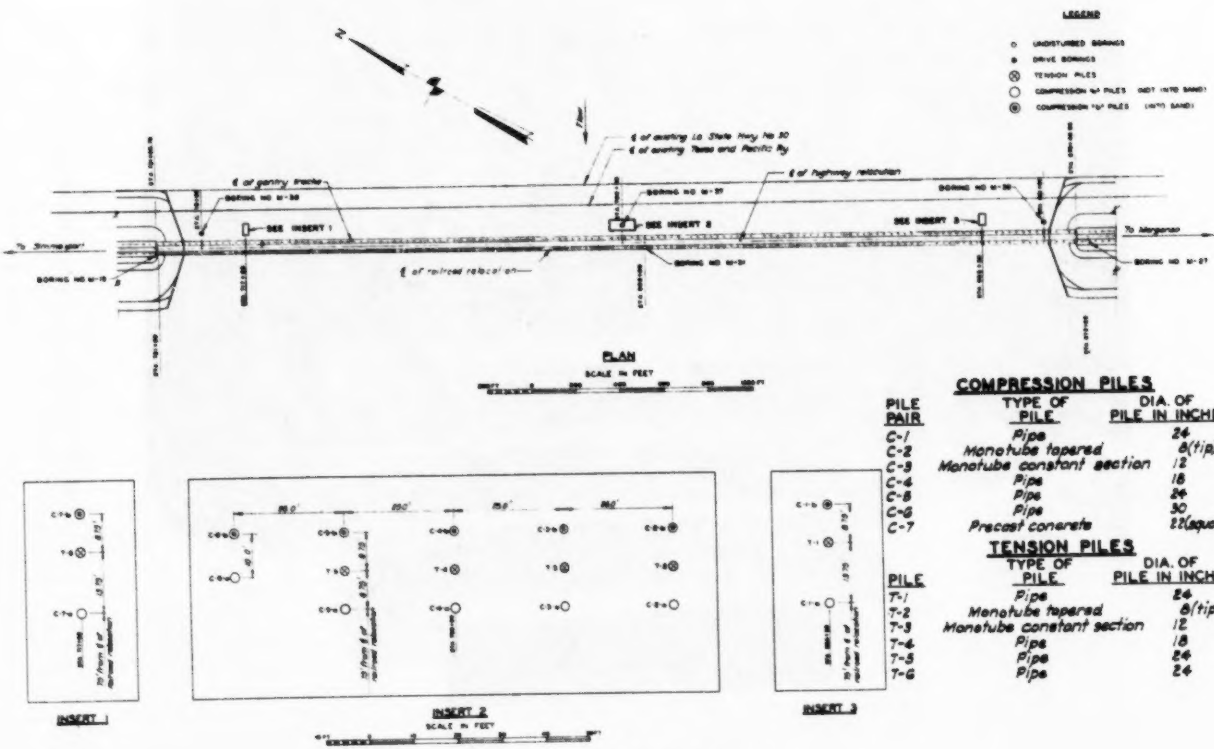
Average cohesive strength of foundation clays as determined by unconfined and quick triaxial tests = 660 lb/sq ft.

Table 6

COMPARISON BETWEEN TEST RESULTS AND PILE DRIVING FORMULAS

Pile	Size and Type	(1) Total Ultimate Test Capacity Tons	(2) Design Short Time (Test) Load F.S. = 2.0	(3) Ultimate Tip Test Capacity Tons	(4) Design Long Time (Test) Load F.S. = 1.5	(5) Engr. News R Tons	(5) Engr. News (Mod. 1) R Tons	(5) Engr. News (Mod. 2) R Tons	(5) Eytelwein R Tons	(5) Canadian National R Tons	(5) Pacific Coast R Tons
C-1-b	24-in. pipe	343	172	165	123	230	243	92	97	45	51
C-2-b	8-in. tip Monotube tapered	162	81	80	53	109	123	45	52	19	21
C-3-b	12-in. Monotube constant section	145	73	90	60	118	183	46	78	18	19
C-4-b	18-in. pipe	244	122	165	110	118	92	46	37	29	29
C-5-b	24-in. pipe	304	152	185	123	230	244	100	101	43	54
C-6-b	30-in. pipe	452	226	280	187	238	204	92	79	48	55
C-7-b	22-in. sq precast concrete	287	144	140	93	230	56	100	19	59	39
(6) Av. Ratio:	Design Formula Capacity / Design Short Time (Test) Load					1.35	1.30	0.55	0.53	0.27	0.28
Av. Ratio:	Design Formula Capacity / Design Long Time (Test) Load					1.80	1.69	0.74	0.69	0.36	0.37
Av. Ratio:	Ultimate Formula Capacity (7) / Total Ultimate Test Capacity					4.05	3.90	1.65	1.59	0.41	0.55
Av. Ratio:	Ultimate Formula Capacity (7) / Ultimate Tip Test Capacity					7.20	6.76	2.96	2.75	0.72	0.95

(1) The total test capacity was taken from "Average Failure Load" - Table 1.
(2) Total ultimate test capacity ÷ 2.0. In other words, the factor of safety against plunging under a "short time" load is 2.0.
(3) The ultimate tip test capacity was determined by inspection of tip load curves.
(4) Ultimate tip test capacity ÷ 1.5. In other words, the factor of safety against detrimental settlement under a "long time" load is 1.5.
(5) Formula capacities indicated by the pile driving formulas are design loads based on the driving resistance for the last 3-in. of driving.
(6) Average ratios were computed by averaging data for individual piles.
(7) Design formula capacity times recommended factors of safety.



COMPRESSION PILES

PILE PAIR	TYPE OF PILE	DIA. OF PILE IN INCHES
C-1	Pipe	24
C-2	Monotube tapered	8(tip)
C-3	Monotube constant section	12
C-4	Pipe	18
C-5	Pipe	24
C-6	Pipe	30
C-7	Precast concrete	22(square)

TENSION PILES

PILE	TYPE OF PILE	DIA. OF PILE IN INCHES
T-1	Pipe	24
T-2	Monotube tapered	8(tip)
T-3	Monotube constant section	12
T-4	Pipe	18
T-5	Pipe	24
T-6	Pipe	24

Figure 1. Test pile layout

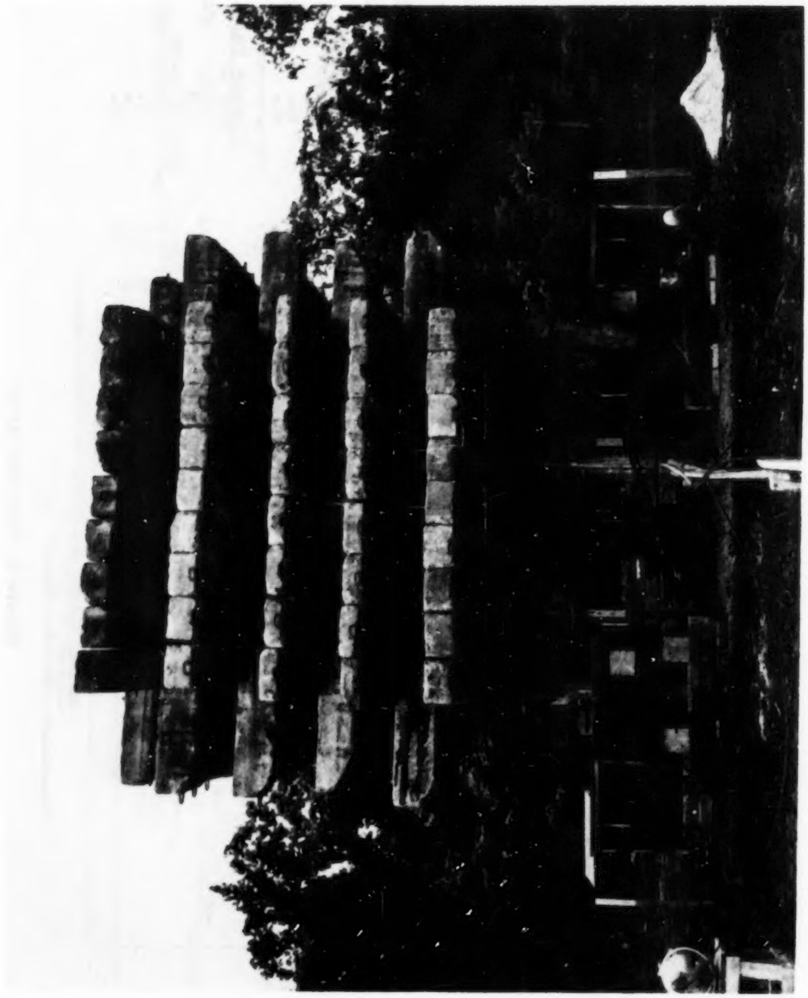


Figure 2. 400 tons over pile C-1-b

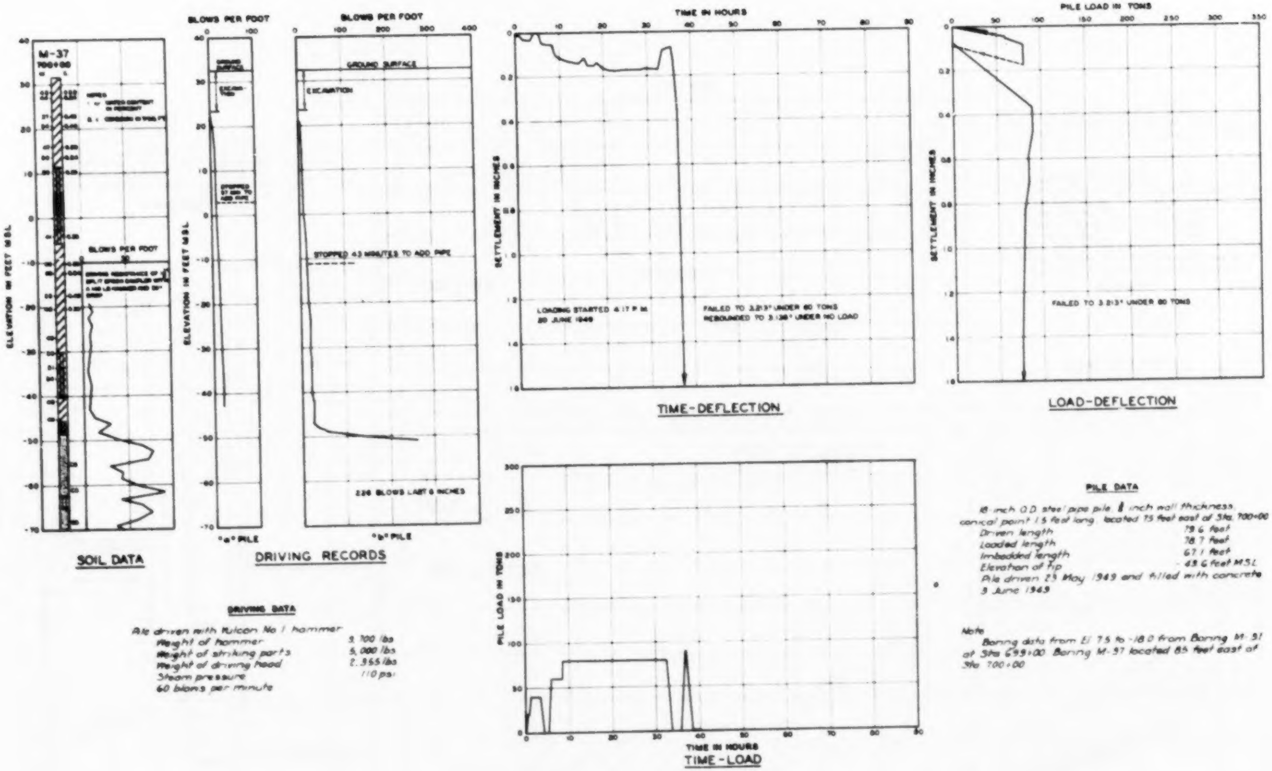


Figure 3. Compression pile test data; pile C-4-a, 18-inch pipe pile

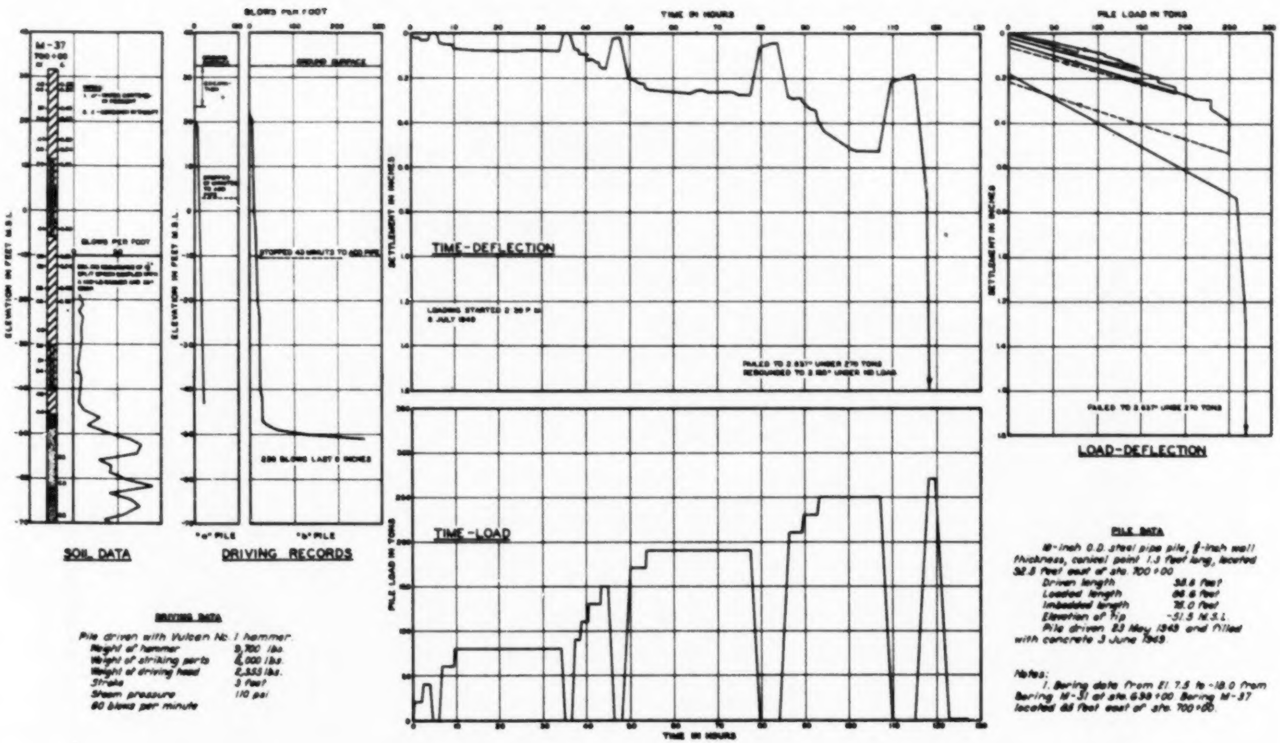


Figure 4. Compression pile test data; pile C-4-b, 18-inch pipe pile

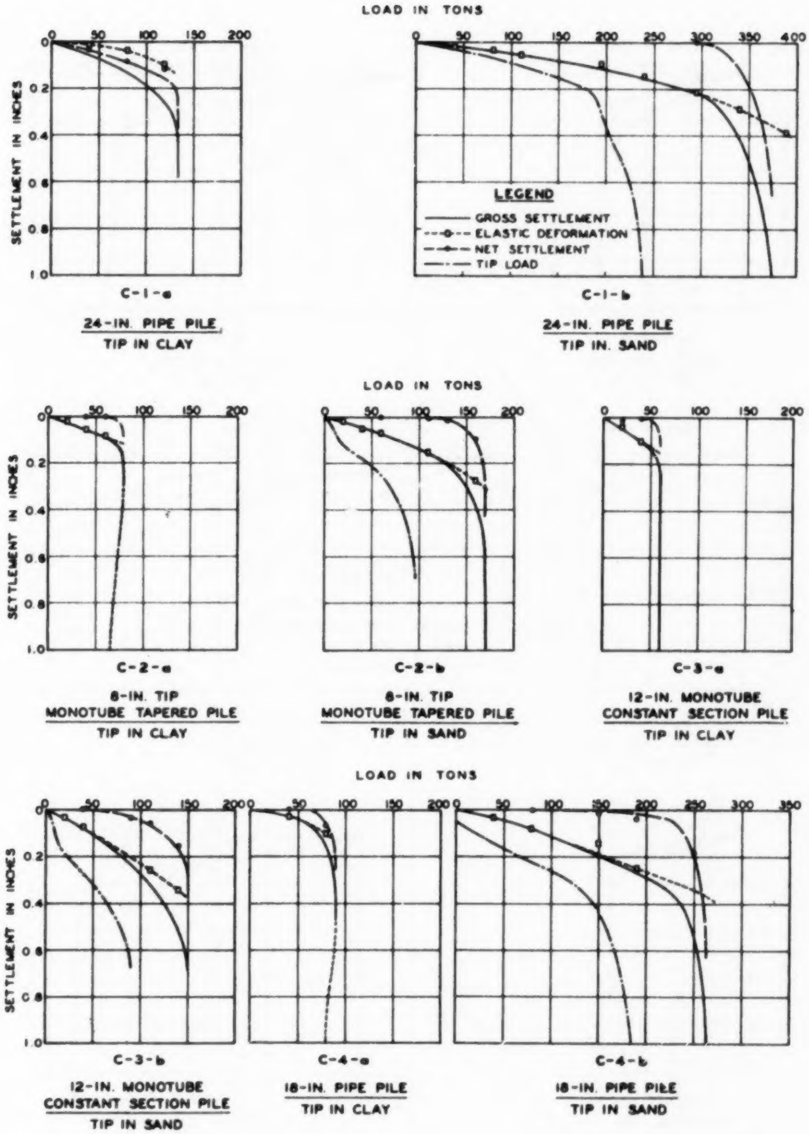


Figure 5. Load settlement curves, piles C-1 thru C-4

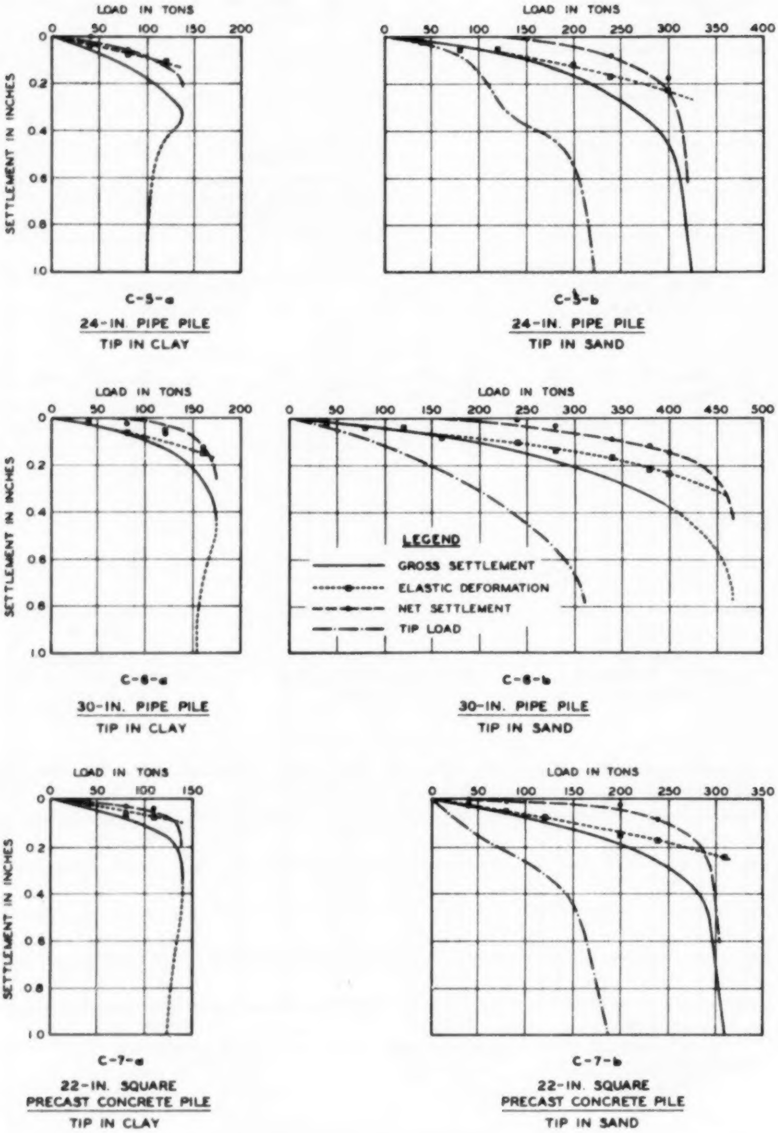


Figure 6. Load settlement curves, piles C-5 thru C-7

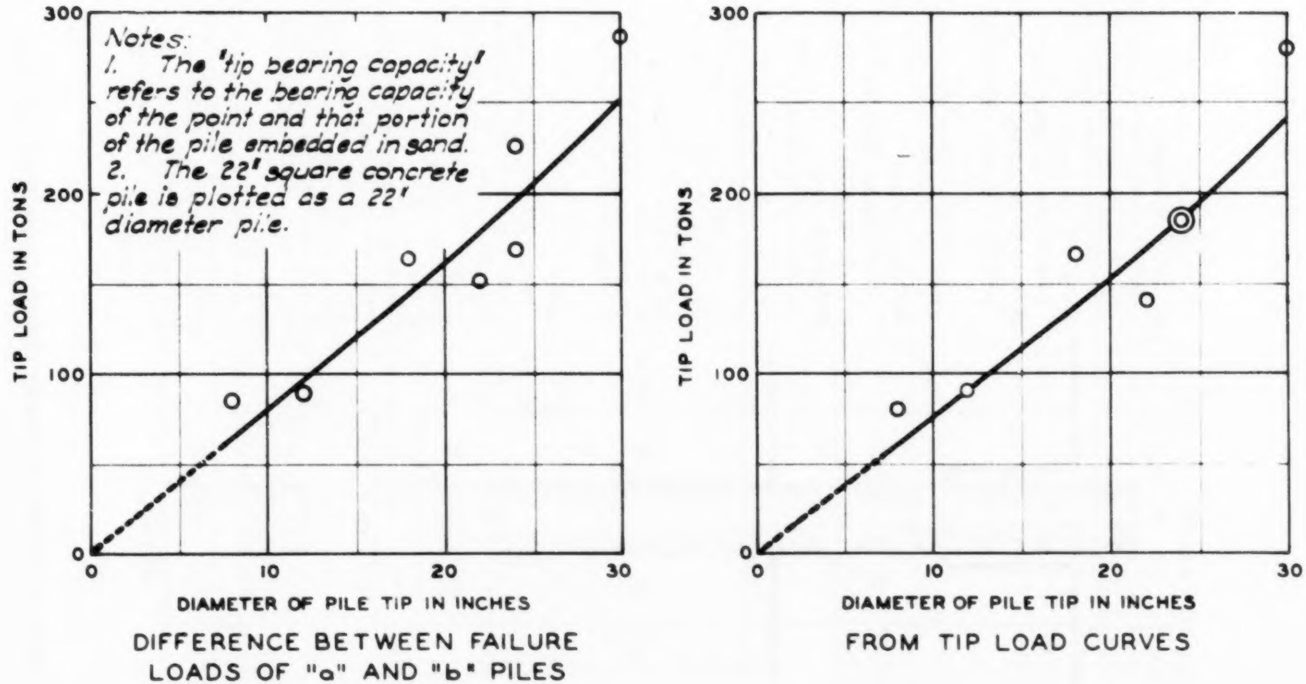
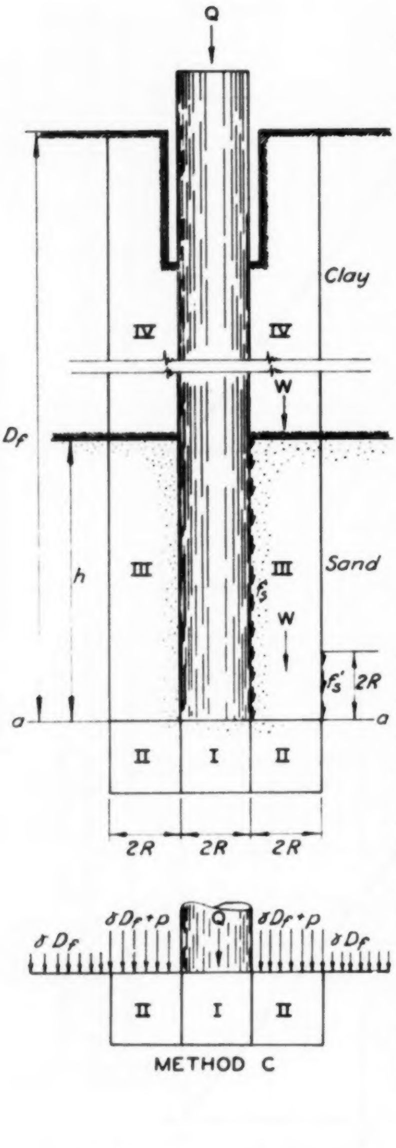


Figure 7. Average tip bearing capacity



Method A, Terzaghi

$$Q = \pi R^2(\gamma D_f N_q + 0.6\gamma R N_\gamma) + 2\pi R h f_s$$

$$f_s = \gamma\left(D_f - \frac{h}{2}\right)\tan\phi$$

γ = Effective unit weight of soil

Method B, Terzaghi

$$Q = \pi R^2(\gamma D_f N_q + 0.6\gamma R N_\gamma)$$

Method C, Terzaghi

$$Q = \pi R^2[(\gamma D_f + p)N_q + 0.6\gamma R N_\gamma] + 2\pi R h f_s$$

$$p = \frac{2\pi R h f_s + 12\pi R^2 f_s'}{8\pi R^2}$$

Method D, Jaky

$$Q = \pi R^2 \gamma D_f \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi \tan\phi} + 2\pi R h f_s$$

Method E, Jaky

$$Q = \pi R^2 \gamma D_f \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi \tan\phi}$$

Q = Total load on pile at failure, clay carrying no load

D_f = Depth of pile point below ground surface.

h = Penetration on pile into sand

γ = Effective unit weight of soil

R = Radius of pile

f_s = Skin friction on pile in sand

Notes:

1. Method A is the equation for cylindrical piers and piles given by Terzaghi and Peck "Soil Mechanics in Engineering Practice", pp 176-177.
2. Method B is a modification of method A.
3. Method C is based on method suggested by Terzaghi in "Theoretical Soil Mechanics", p 134 and in discussion of paper "Appl. of Soil Mech. in Designing Bldg. Fnd.", Trans., ASCE, vol. 109(1944), p 430.
4. Method D is equation given by Jaky in paper "On Bearing Capacity of Piles" Second Int. Conf. on Soil Mech., vol. 1 p 102.
5. Method E is a modification of method D.

Figure 8. Theoretical bearing capacity of pile driven into sand

Notes:

1. Failure loads (•) from tip load curves corrected for additional length of "b" piles in clay and adjusted to a 5 ft penetration into sand and an 85 ft depth of the point on basis of Terzaghi A equation.

2. Curves computed by equations on figure 8 for $\phi = 30°$, a 5 ft penetration into sand, and an 85 ft depth of the point.

3. 7.6 ⚲ Adjustment from actual failure load to that computed for a 5-ft penetration into sand. Number is actual penetration of pile into sand.

4. Tip load for 30-in. pile based on extrapolation of gross load curve.

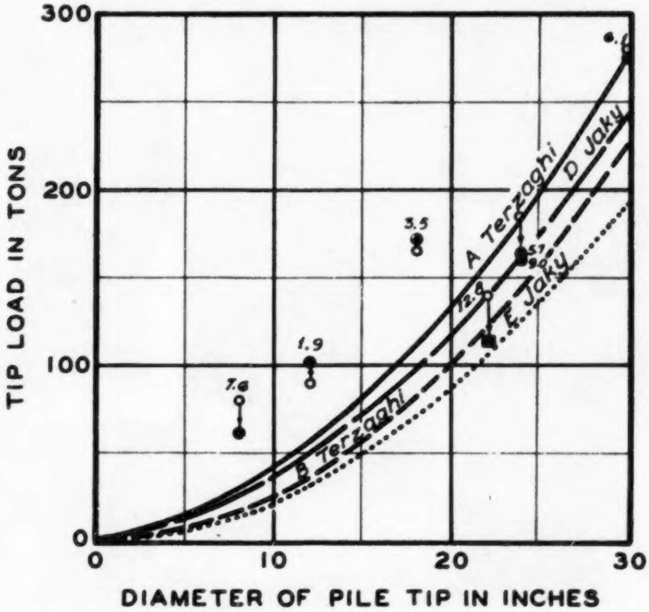


Figure 9. Adjusted tip bearing capacity

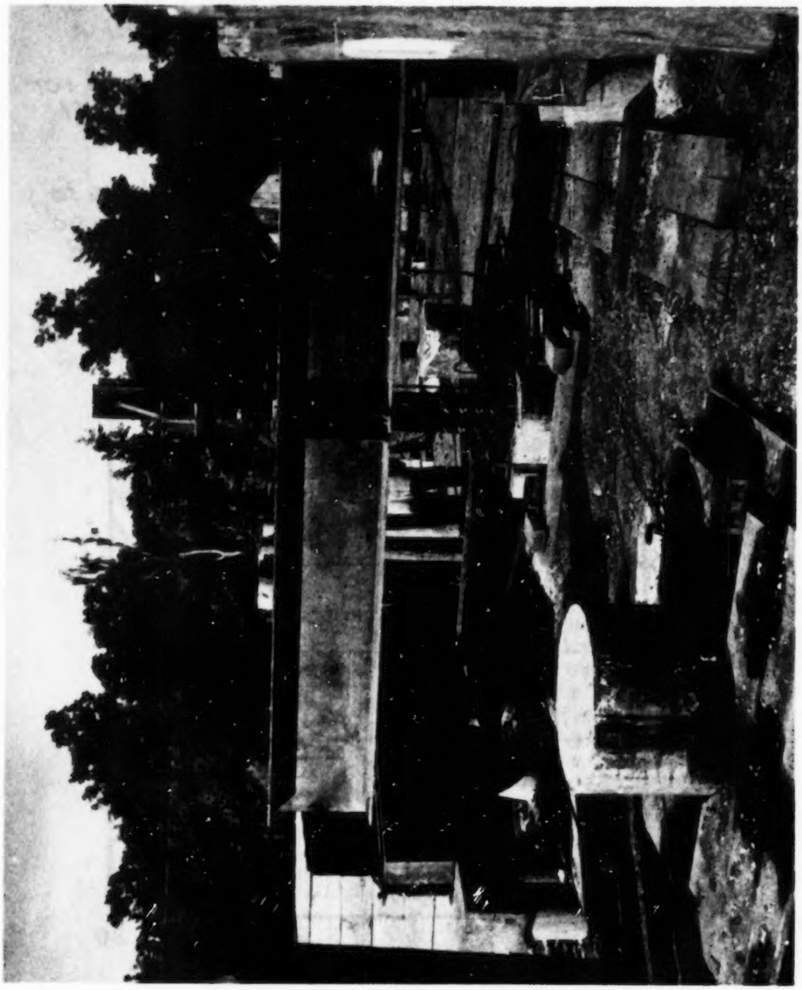


Figure 10. Tension test on pile T-1

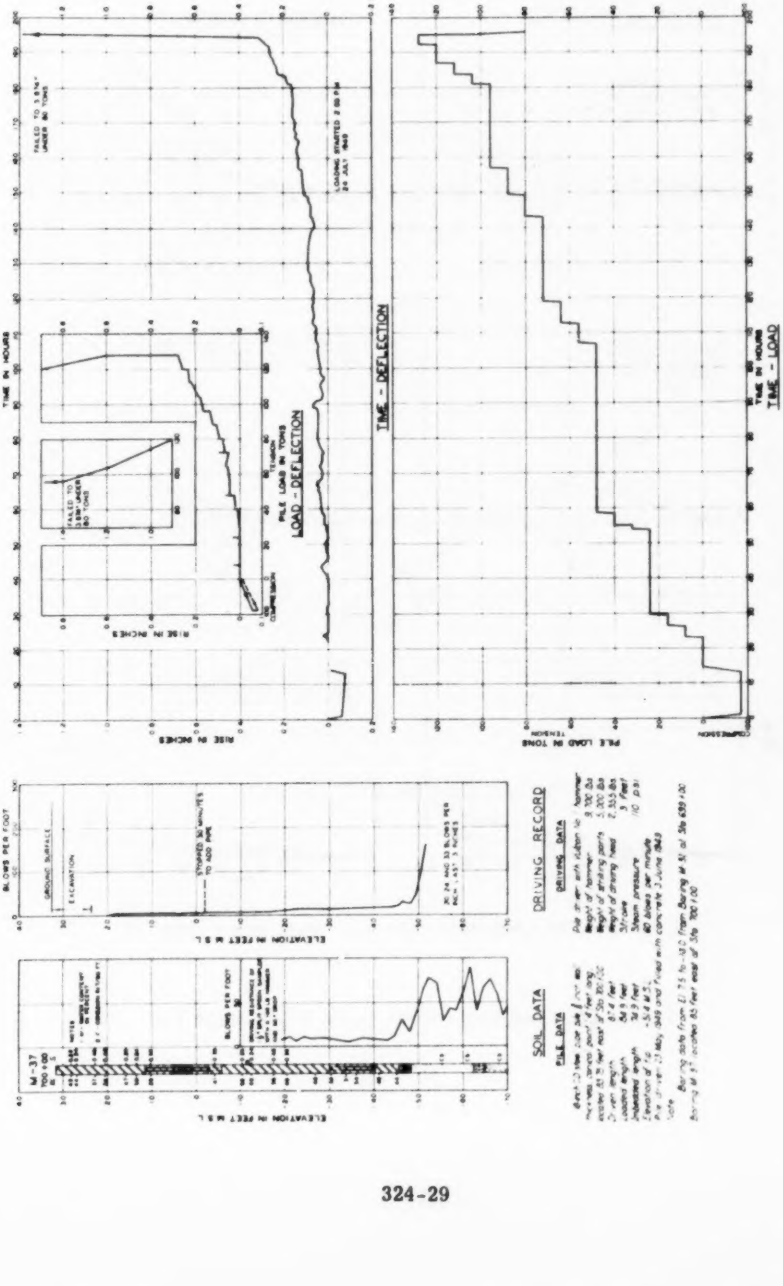


Figure 11. Tension pile test data, pile T-4, 18-inch pipe pile

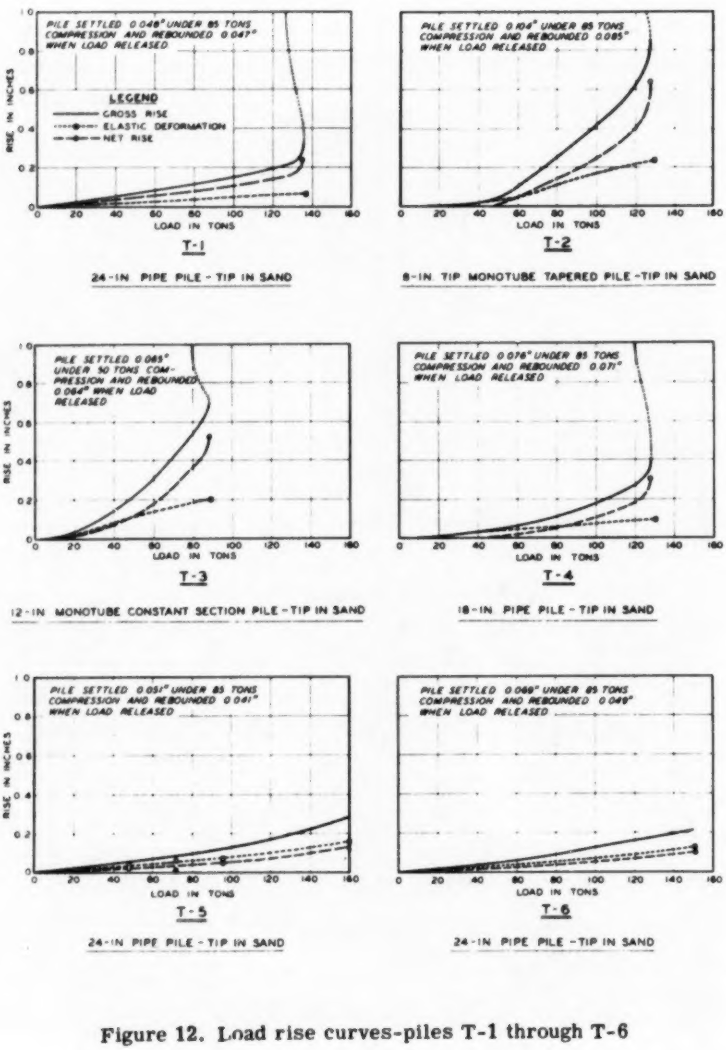


Figure 12. Load rise curves-piles T-1 through T-6

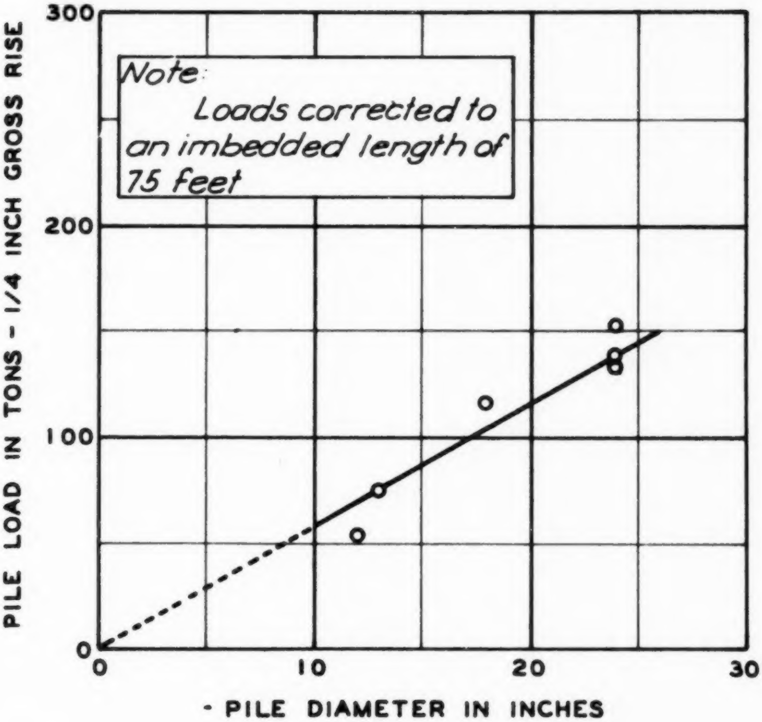


Figure 13. Average tension capacity